CHAPTER 6

REINFORCED MASONRY WALLS

6-1. Introduction. This chapter covers the design of reinforced masonry walls by the working stress method for lateral out-of-plane loads and axial loads. The design of reinforced masonry walls for in-plane lateral loads (shear walls) and axial loads is covered in chapter 7. General design criteria, section properties, and allowable stresses used but not contained herein are covered in chapter 5.

6-2. Design loadings.

a. Lateral loads. Lateral out-of-plane loads on masonry walls are determined from wind forces as given in TM 5-809-1/AFM 88-3, Chapter 1 or from seismic forces as given in TM 5-809-10/NAVFAC P-355/AFM 88-3, Chapter 13.

b. Axial loads. Vertical in-plane compression or tension loads on masonry walls are determined from dead, live, snow, and wind uplift forces as given in TM 5-809-1/AFM 88-3, Chapter 1.

6-3. Structural behavior.

a. Lateral loads. Most masonry walls are designed to span vertically and transfer the lateral loads to the roof, floor or foundation. Normally, the walls are designed as simple beams spanning between structural supports. Simple beam action is assumed even though reinforcement, which is needed to control horizontal flexural cracking at the floor levels or to provide connectivity, may be present and will provide at least partial continuity. Under certain circumstances, such as when a system of pilasters is present, the masonry walls may be designed to span horizontally between pilasters which in turn span vertically to transfer the lateral loads to the horizontal structural support elements above and below.

b. Axial loads. Loads enter the wall from roofs, floors, or beams and are transferred axially to the foundation. When the resultant axial force is tension from wind uplift loadings, mortar tension will not be used to resist these uplift forces. Instead, adequate reinforcement will be provided to anchor the top of wall bond beam to the remainder of the wall and on down to the foundation. If the resultant of the vertical loads which are applied to the wall at any level is not at the center of the wall; that is, it is not concentric; due allowance will be made for the effects of eccentric loading. This includes any moments that are due to eccentric loading as well as any additional moments caused by the rotation of floor or roof elements that frame into the wall.

(1) Uniform loads. Uniform loads enter the wall as line loads, stressing the wall uniformly along its length.

(2) Concentrated loads. When concentrated loads are not supported by structural elements, such as pilasters, they may be distributed over a length of wall equal to the width of bearing plus four times the wall thickness, but not to exceed the center to center distance between concentrated loads. Concentrated loads will not be distributed across control joints.

c. Combined loads. The combined effects of lateral and axial loads may be assumed to act according to the straight-line interaction equations given in this chapter or may be combined by other methods which are based on accepted principles of mechanics.

6-4. Wall design equations. The equations in this paragraph may be used for the design of walls subjected to bending and axial loads. Lateral (wind or seismic) loading will be applied inward and outward on all exterior walls. Both the condition where the moment due to wind loading and the moment due to axial load eccentricity are additive and the condition where they are not additive are shown on figures 6-1 and 6-2, respectively.

a. Bending equations. The horizontal reaction at the bottom of the wall due to the combined effects of eccentric and lateral loads is "R_a" and is determined as follows:

$$R_a = \frac{wh}{2} \pm \frac{P_e}{12h} \text{ (lb/ft of wall)} \qquad \text{(eq 6-1)}$$

Where:

P = The axial load, pounds per foot of wall length.

e = the distance from the centerline of the wall to the load P, inches.

h = The height of the wall, feet.

w = The lateral load on the wall, psf.

Note: The "±" in equation 6-1 refers to the two conditions; (1) where the eccentric and lateral loads are additive, and (2) where the eccentric and lateral loads are *not* additive. Both conditions will occur on all exterior walls.

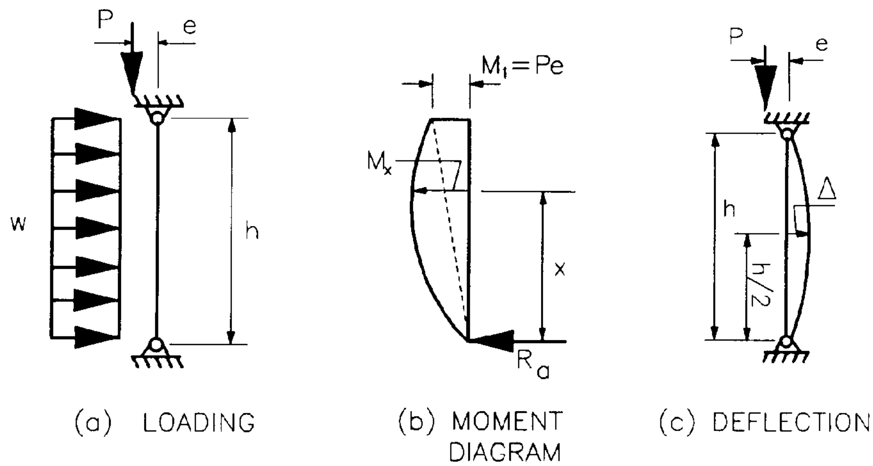

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Figure 6–1. Wall loading, moment, and deflection diagram—Wind and axial load moments additive.

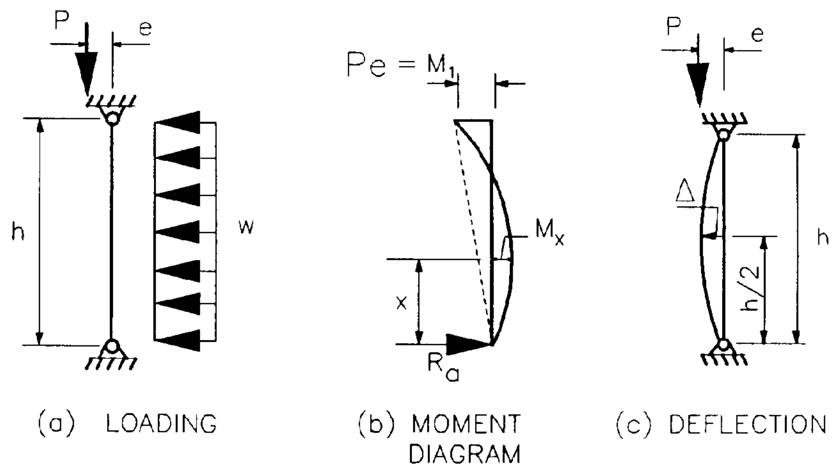

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Figure 6–2. Wall loading, moment, and deflection diagrams—Wind and axial load moments not additive.

The moment at a distance “x” feet from the bottom of the wall is “Mx” and is determined as follows:

$$M_x = R_sx - \frac{wx^2}{2} \text{ (lb-ft/ft)} \qquad \text{(eq 6-2)}$$

If R_a in equation 6-2 for the additive condition is replaced with its equivalent from equation 6-1, equation 6-2 becomes:

$$M_x = \frac{Pex}{12h} + \frac{whx}{2} \text{ (lb-ft/ft)} \qquad \text{(eq 6-3)}$$

This M_x equation can then be simplified to:

$$M_x = (x)\left[\frac{Pe}{12h} + \frac{w(h-x)}{2}\right] \text{ (lb-ft/ft)} \qquad \text{(eq 6-4)}$$

When the spacing between reinforcing bars, S, in inches, is included the equation becomes:

$$M_x = \frac{S_x}{12}\left[\frac{Pe}{12h} + \frac{w(h-x)}{2}\right] \text{ (lb-ft/S)} \qquad \text{(eq 6-5)}$$

(1) When w = 0 and P is eccentric, the maximum bending moment occurs at the top of the wall where x = h and equation 6-5 becomes:

$$M_{max} = \frac{SPe}{(12)(12)} \text{ (lb-ft/S)} \qquad \text{(eq 6-6)}$$

(2) When w > 0 and P is *not* eccentric, the maximum moment occurs at mid-height of the wall where x = h/2 and equation 6-5 becomes:

$$M_{max} = \frac{Swh^2}{(12)(8)} \text{ (lb-ft/S)} \qquad \text{(eq 6-7)}$$

(3) When w > 0, P is eccentric, and the moments due to "w" and "Pe" are additive; the location of the maximum moment can be determined by differentiating the moment equation with respect to x, setting the equation equal to zero, and then solving for x. By performing this operation on equation 6-3, the "x" location where the maximum moment occurs can be determined as follows—

$$\frac{dM_x}{dx} = \frac{wh}{2} + \frac{Pe}{12h} - wx = 0$$

Solving for x;

$$x = \frac{h}{2} + \frac{Pe}{12wh} \text{ (ft)} \qquad \text{(eq 6-8)}$$

It should be reiterated that this maximum moment condition will occur only when the moment due to the eccentricity of the axial loads and the moment due to the lateral load are additive. Substituting equation 6-8 into 6-2, the maximum moment, per length of wall equal to reinforcing bar spacing, 5, can be found as follows:

$$M_{max} = \frac{S(R_a)^2}{2w} \text{ (ft-lbs/S)} \qquad \text{(eq 6-9)}$$

Equations similar to 6-3 through 6-9 can be similarly derived for the case when the moment due to lateral loading and the moment due to eccentric axial loading are not additive.

b. Axial compression equations. The axial stress at any height, h, in a wall is determined as follows:

$$f_a = \left[\frac{P + w_2(h - x)}{A_e}\right] \text{ (psi)} \qquad \text{(eq 6-10)}$$

Where:

w_2 = The weight of the wall, psf.
A_e = The effective area of the wall, in^2/ft.

(1) When x = h (top of wall), there is no wall weight and equation 6-10 becomes:

$$f_a = \frac{P}{A_e} \text{ (psi)} \qquad \text{(eq 6-11)}$$

(2) When x = 0 (bottom of wall) the entire wall weight is included and equation 6-10 becomes:

$$f_a = \frac{P + w_2h}{A_e} \text{ (psi)} \qquad \text{(eq 6-12)}$$

c. Combined stresses.

(1) In walls subject to combined axial compression and flexural stresses, the masonry will be designed in accordance with the interaction equations as follows—

$$\left[\frac{f_a}{F_a} + \frac{f_b}{F_b}\right] \text{ OR } \left[\frac{f_a}{F_a} + \frac{M_x}{M_{rm}}\right] \leq 1.00 \qquad \text{(eq 6-13)}$$

Since a 33% overstress is allowed when wind or seismic loads are considered, the allowable stresses and resisting moment in equation 6-13 may be increased by 33% or interaction equation 6-14 may be used.

$$\left[\frac{f_a}{F_a} + \frac{f_b}{F_b}\right] \text{ OR } \left[\frac{f_a}{F_a} + \frac{M_x}{M_{rm}}\right] \leq 1.33 \qquad \text{(eq 6-14)}$$

(2) In walls subject to combined axial and flexural stress, the reinforcing steel will be designed using interaction equations as follows:

$$\left[\frac{M_x}{M_{rs}} - \frac{f_a}{F_a}\right] \leq 1.00 \qquad \text{(eq 6-15)}$$

Since a 33% overstress is allowed when wind or seismic loads are considered, the allowable stress and resisting moment in equation 6-15 may be increased by 33% or interaction equation 6-16 may be used.

$$\left[\frac{M_x}{M_{rs}} - \frac{f_a}{F_a}\right] \leq 1.33 \qquad \text{(eq 6-16)}$$

Note that when the reinforcing steel is being checked, the minimum axial stress, ~a, must be used. Note also that it is conservative to *not* consider axial loading ($f_a = 0$) when checking the reinforcing steel stress.

d. Shear equations. The shear stress at the bottom of the wall is determined by the following equation:

$$f_v = \frac{R_a}{b_w d} \text{ (psi)} \qquad \text{(eq 6-17)}$$

Where:

b_w = The width of the masonry element effective in resisting out-of-plane shear as given in chapter 5, inches.

d = The depth of the masonry element effective in resisting shear, given "d_1" for one reinforcing bar per cell and "d_2" for two bars per cell in chapter 5.

6-5. P-delta effect. The "P-delta effect" is the increase in moment and deflection resulting from multiplying the mid-height defection of a wall (due to lateral and eccentric loadings as discussed above) by the summation of the axial load, P, at the top of the wall and the weight of the top half of the wall. When the height to nominal thickness ratio of the wall is less than 24, the "P-delta effect" is minor and may be neglected. For walls where the height to nominal thickness ratios is greater than 24, the mid-height deflection, Δs, will be computed as follows:

When $M_{mid} < M_{cr}$;

$$\Delta_s = \frac{(5)(M_{mid})(h^2)(144)}{(48)(E_m)(I_g)} \text{ (in)} \qquad \text{(eq 6-18)}$$

When $M_{cr} < M_{mid} < M_r$;

$$\Delta_s = \frac{(5)(M_{cr})(h^2)(144)}{(48)(E_m)(I_g)} + \frac{(5)(M_{mid} - M_{cr})(h^2)(144)}{(48)(E_m)(I_{cr})} \text{ (in)} \qquad \text{(eq 6-19)}$$

Where:

h = The wall height, feet

M_{mid} = The moment at the mid-height of the panel, including the "P-Delta effect", inch-pounds.

E_m = The modulus of elasticity, psi = $1000f'_m$

I_g = The gross moment of inertia of the wall cross section, in^4.

I_{cr} = The cracked moment of inertia of the wall cross section, in^4

M_{cr} = The cracking moment strength of the masonry wall, inch-pounds.

M_{rm} = The allowable resisting moment of the masonry wall, inch-pounds.

6-6. Walls with openings. Walls at the edge of openings or between openings are required to resist additional tributary axial and lateral loads. The additional tributary axial loads are due to the weight of masonry above the opening and vertical loads applied to the tributary masonry above the opening. The additional tributary lateral loads are the lateral loads on non-masonry wall components (doors, windows, etc.) that are laterally supported by the adjacent masonry wall elements. The tributary load area width will be measured from the centerline of the openings. Masonry wall elements between and alongside openings that are subjected to combined loading will be designed in accordance with equations 6-13 through 6-16. Due allowance will be made for eccentricity.

6-7. Design aids. Appendix B contains design aids that may be used in the design of reinforced masonry walls. Tables B-1 through B-14 provide the properties of wall stiffeners with varying reinforcement (size, spacing and number of bars per cell), varying wall thickness (6, 8, 10, and 12 inch nominal thickness) and two mortar types (S and N). Tables B-15 through B-50 provide reinforcing steel sizes and spacings for varying wall heights, lateral loads, wall thicknesses, axial loads (with and without eccentricity), using type S mortar.

6-8. Design examples. The following design examples illustrate the development and use of the design aids in Appendix B.

a. Design example 1. This illustrative example considers only one combination of wind and eccentric axial loading. When performing a complete wall design, all appropriate load combinations must be considered.

(1) *Given—*

(a) 12-inch CMU loadbearing wall

(b) Wall height (h) = 24 ft

(c) Lateral wind load (w) = 25 lb/ft^2

(d) Axial load (P) = 1500 lb/ft

(e) Eccentricity (e) = 0.5t, in
(f) The moments due to lateral wind load and to axial eccentricity are additive.
(g) $f'_m = 1{,}350$ lb/in^2
(h) $F_m = (0.33)f'_m = 450$ lb/in^2
(i) $E_m = 1000f'_m = 1{,}350{,}000$ lb/in^2
(j) $F_s = 24{,}000$ lb/in^2
(k) $E_s = 29{,}000{,}000$ lb/in^2

(l) $$n = \frac{E_s}{E_m} = \frac{29{,}000{,}000}{1{,}350{,}000} = 21.5$$

(2) *Problem—*

(a) Determine the reinforcing bar size and spacing required to resist the given loadings.

(b) Compare the calculated resisting moment values with the values for resisting moments given in table B-4.

(c) Compare the reinforcing results from the calculated solution with the direct solution given in table B-47.

(3) *Solution.* Equations are from chapters 5 and 6. Flexural Check:

(a) First determine the maximum applied moment that must be resisted by the wall.

Horizontal reaction at the bottom of the wall is R_a:

$$R_a = \frac{P_e}{12h} + \frac{wh}{2}$$

$$= \frac{(1500\ \text{lb/ft})[(0.5)(11.625\ \text{in})]}{(12\ \text{in/ft})(24\ \text{ft})} + \frac{(25\ \text{lb/ft}^2)(24\ \text{ft})}{2}$$

= 30.3 + 300.0 = 330.3 lb/ft of wall

Location where maximum moment occurs is “x” distance from the bottom of the wall:

$$M_x = R_a x - \frac{wx^2}{2}$$

$$= (330.3\ \text{lb})(x) - \frac{(25\ \text{lb/ft}^2)(x^2)}{2}$$

Differentiating with respect to x;

$$\frac{dM_x}{d_x} = R_a - wx = 330.3 - 25x = 0$$

Solving for x;

$$x = \frac{330.3}{25} = 13.2\ \text{ft bottom of wall}$$

Maximum moment in the wall is M_{max}

$$M_{max} = (330.31b)(13.2\ \text{ft}) - \frac{(25\ \text{lb/ft}^2)(13.2\text{ft})^2}{2}$$

= 4360 - 2178 = 2182 ft-lb/foot of wall

Assume the reinforcement spacing, S, is 24 inches and determine the design maximum moment, Design M_{max}, in the wall as follows:

Design M_{max} = (2182 ft-lbX24 in)/(12 in/ft)
= 4364 ft-lb/S

(b) Determine the resisting moments n the wall assuming 1-#6 @ 24 in. o.c. Assume the flexural compression area is rectangular and compare to the T-section design from table B-4.

Masonry resisting moment is M_{rm}:

$$M_{rm} = \frac{F_m kjbd^2}{2(12)}$$

Where:

$p = A_s/bd_1 = (0.44\ \text{in}^2)/(24\ \text{in})(5.81\ \text{in}) = 0.0032$
$np = (21.5)(0.0032) = 0.0688$
$k = [(np)^2 + 2np]^{½} - np$
$k = [(0.0688)^2 + (2)(0.0688)]^{½} - 0.0688$
$= [0.00473 + 0.1376]^{½} - 0.0688 = 0.308$
$j = 1 - k/3 = 1 - 0.308/3 = 0.897$

$kd = 0.308(5.81 \text{ in}) = 1.79 \text{ in}$

Note that kd is greater than the face shell thickness, therefore the actual design section would be a T-section. The following will show that the difference generated by assuming a rectangular section is negligible.

$$M_{rm} = \frac{(450 \text{ lb/in}^2)(0.308)(0.897)(24)(5.81 \text{ in})^2}{2(12)}$$

$= 4{,}196 \text{ ft-lb/S} \approx 4{,}147 \text{ ft-lb/S}$ (table B-4)

Reinforcing steel resisting moment is M_{rs}:

$$M_{rs} = \frac{F_s A_s jd}{12}$$

$$M_{rs} = \frac{(24{,}000 \text{ lb/in}^2)(0.44 \text{ in}^2)(0.897)(5.81)}{12}$$

Note that the difference between the T-section analysis moments from table B-4 and the computed rectangular section moments is negligible (approximately 1%).

(c) To illustrate the derivation of the table values, a T-section analysis will be performed.

$$k_T = \frac{np + ½(t_s/d)^2}{np + (t_s/d)}$$

$$= \frac{(21.5 \times 0.0032) + 1/2(1.5/5.81)^2}{(21.5 \times 0.0032) + (1.5/5.81)}$$

$= 0.312$

$$j_T = \frac{6 - 6(t_s/d) + 2(t_s/d)^2 + (t_s/d)(1/2pn)}{6 - 3(t_s/d)}$$

$$j_T = \frac{6 - [6(1.5/5.81)] + 2(1.5/5.81)^2}{6 - 3(1.5/5.81)}$$

$$+ \frac{(1.5/5.81)[1/2 \times 0.0032 \times 21.5)}{6 - 3(1.5/5.81)} = 0.902$$

$$M_{rsT} = \frac{A_s F_s j_T d}{12}$$

$$M_{rsT} = \frac{(0.44 \text{ in}^2)(24{,}000 \text{ lb/in}^2)(0.902)(5.81)}{12}$$

$= 4611 \text{ ft-lbs} \approx 4603 \text{ ft-lbs}$ (table B-4)

$$M_{rsT} = \frac{450[1 - 1.5/(2 \times 0.312 \times 5.81)](24)(1.5)(0.902)(5.81)}{12}$$

$= 4147 \text{ ft-lbs} \approx 4147 \text{ ft-lbs}$ (table B-4)

Note that since wind loadings are a part of the loading combination, the resisting moments of the wall cross section may be increased by 33%. Thus, the design resisting moments for the masonry and the reinforcing steel, respectively are:

$M_{rmT} = 1.33(4147 \text{ ft-lb/S}) = 5{,}516 \text{ ft-lb/S}$

$M_{rsT} = 1.33(4611 \text{ ft-lb/S}) = 6{,}133 \text{ ft-lb/S}$

Note: The masonry resisting moment controls the design:

$M_{rmT} = 5{,}516 \text{ ft-lb/S} > M_{max} = 4{,}364 \text{ ft-lb/S}$ ....O.K.

Axial Load Check: For the 12-inch CMU wall with reinforcing spaced at 24 inches o.c., the effective area in compression, A_e, is 68 in^2/ft and the weight of the wall, W_2, is 102 lb/ft^2.

The axial compressive stress in the wall, f_a, is determined as follows:

$$f_a = \frac{P + (w_2)(h - x)}{A_e}$$

$$f_a = \frac{(1500 \text{ lb}) + (102 \text{ lb/ft}^2)(24 \text{ ft} - 13.2 \text{ ft})}{68 \text{ in}^2} = 38.3 \text{ lb/in}^2$$

The allowable axial compressive stress in wall is F_a:

$$F_a = 0.2f'_m\left[1 - \left[\frac{12h}{40t_n}\right]^3\right]$$

$$F_a = (0.2)(1350)\left[1 - \left[\frac{(12)(24)}{(40)(12)}\right]^3\right] = 212.0\ \text{lb/in}^2$$

$f_a = 38.3$ lb/in^2 < $F_a = 212.0$ lb/in^2 ...O.K.

Combined Load Check: Since the masonry resisting moment controls, only the masonry need be checked in the combined stress condition. The unity equation will be used. Since wind loadings 'are a part of the loading combination, the allowable axial compressive stress, F_a, may be increased by 33%.

$F_a = 1.33(212.0$ lb/in$^2) = 282.0$ lb/in^2

$$\frac{f_a}{F_a} + \frac{M_{max}}{M_r} \leq 1.0$$

$$\frac{38.3\ \text{lb/in}^2}{282.0\ \text{lb/in}^2} + \frac{4364\ \text{ft-lb}}{5516\ \text{ft-lb}} = 0.14 + 0.79 \leq 1.00$$

...O.K.

Direct solution (table B-47): Using the design parameters given above; the 1-#6 bar spaced at 24 inches o.c. which was determined by the design calculations; is sufficient reinforcement.

(4) *Summary*. 1-#6 bar per cell spaced at 24 inches o.c. is sufficient.

b. Design example 2. This illustrative example considers only one combination of wind and eccentric axial loading. When performing a complete wall design, all appropriate load combinations must be considered.

(1) *Given*:

(a) 12-inch CMU wall
(b) Wall height (h) = 19'-4"
(c) Lateral wind load (w) = 22 lb/ft^2
(d) Axial load (P) = 650 lb/ft
(e) Eccentricity (e) = t/3
(f) $f'_m = 1350$ lb/in^2
(g) $F_s = 24{,}000$ lb/in^2

(2) *Problem*. Find the required spacing of #6 bars using the tables in appendix B and linear interpolation.

(3) *Solution*. Interpolating for wall height, axial loading, and wind loading.

Table B–40; P = 500 lb/ft, e = t/3

Wall Ht(ft)	Wind, lb/ft^2 20	22	25
20	56	52.8	48
19.33		57.1	
18	72	65.6	56

Table B–43, P = 1000 lb/ft, e = t/3

Wall Ht(ft)	Wind, lb/ft^2 20	22	25
20	56	49.6	40
19.33		53.3	
18	64	60.8	56

$S_{max} = 57.1 - (57.1 - 53.3)[(650 - 500)/500] = 56$ inches o.c.

(4) *Summary*. The wall will resist the given loading with #6 bars spaced at 56 inches on center.

c. Design example 3. This illustrative example considers only one combination of wind and eccentric axial loading. When performing a complete wall design, all appropriate load combinations must be considered.

(1) *Given*.

(a) 8-inch CMU wall
(b) Wall height (h) = 16 ft
(c) Wall length (L) = 30 ft

(d) Lateral wind load (w) = 30 lb/ft^2. The wind load is positive (inward) on the exterior face of the wall.

(e) Axial load (P) = 300 lb/ft.

(f) Eccentricity (e) = 1 inch. The axial load is applied on the interior side of the wall center line causing a condition where the eccentric and lateral load moments are *not* additive.

(g) f'_m = 1350 lb/in^2

(h) F_s = 24,000 lb/in^2

(i) As shown in figure 6-3, a 12 feet wide by 14 feet high door is located in the wall panel. One edge of the door is 6 feet 8 inches from the wall corner.

(2) *Problem.* Design a stiffener at the door jamb that will resist the applied lateral and axial loads.

(3) *Solution.* Assume that the edge stiffener resists the wind load between the middle of the door and the middle of the 6'-8" wall panel. Also, assume that the stiffener resists the wall weight and the axial load to the middle of the door plus the width of the stiffener.

(a) Determine the wind load, W_L, on the edge stiffener.

$W_L = w \times L_x$

Where:

L_w = The tributary width of the load to the jamb, feet.

$$W_L = (30 \text{ lbs/ft}^2)\left[\frac{12 \text{ ft} + 6.67 \text{ ft}}{2}\right] = 280 \text{ lb/ft}$$

(b) Determine the moment resulting from the eccentricity of the axial load, M_{ecc}.

$M_{ecc} = PL_pe$

Where:

L_p = The distance from the edge of the stiffener to the center line of the door, feet.

$$M_{ecc} = \frac{(300 \text{ lbs/ft})(1.33 \text{ ft} + 6 \text{ ft})(1 \text{ in})}{12 \text{ in/ft}} = 183 \text{ ft-lbs}$$

(c) Determine the distance, x, from the bottom of the wall to where the maximum moment in the edge stiffener occurs.

$$x = \frac{h}{2} - \frac{PL_pe}{W_Lh}$$

$$x = \frac{16 \text{ ft}}{2} - \frac{183 \text{ ft-lbs}}{(280 \text{ lb/ft})(16 \text{ ft})} = 7.96 \text{ ft}$$

(d) Determine the maximum moment in the edge stiffener, M_{max}, as follows:

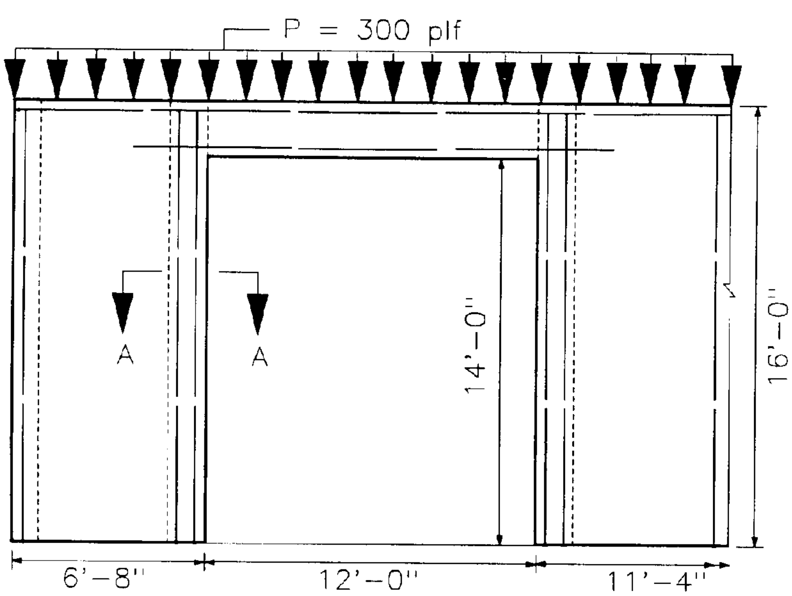

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Figure 6–3. Example 3 wall elevation.

$$M_{max} = R_a x - \frac{w_L x^2}{2}$$

Where:

R_a = The reaction at the bottom of the wall, pounds.

$$= \frac{W_L h}{2} - \frac{PL_P e}{h} = \frac{(280 lb/ft)(16 ft)}{2} - \frac{(183 ft\text{-}lbs)}{16} = 2229 \text{ lbs}$$

$$M_{max} = (2229)(7.96) - \left[\frac{(280)(7.96)^2}{2} \right] = 8872 \text{ ft=lbs}$$

(e) Determine the axial load stress on the edge stiffener, f_a. Assume the lintel over the door is fully grouted from the top of the opening to the top of the wall.

$$f_a = \frac{W_3(h - x)L_s + R_L}{L_s t}$$

Where:

w_3 = The weight of the stiffener, lbs/ft.
L_s = The width of the stiffener, feet.
t = The thickness of the stiffener, inches.
R_L = The lintel reaction, lbs.
= $(P)(L_p) + (w_2)(d)(L_L/2)$

Where:

w_2 = The weight of masonry above the opening, psf.
d = The height of the lintel, feet.
L_L = The length of the lintel, feet.
R_L = (300 lb/ft x 7.33 ft) + (92 lbs/ft^2)(2 ft)(12 ft/2)
= 3303 lbs

$$f_a = \frac{(140 \text{ lbs/ft})(16 \text{ ft} - 7.96 \text{ ft})(1.3 \text{ ft}) + 3303 \text{ lbs}}{(1.3 \text{ ft})(12 \text{ in/ft})(11.62 \text{ in})}$$

= 26.3 lbs/in^2

(f) Determine the allowable axial stress, F_a.

$$F_a = 0.20 f'_m \left[1 - \left[\frac{12h}{40t} \right]^3 \right]$$

$$= (0.20)(1350 \left[1 - \left[\frac{(12)(16)}{(40)(12)} \right]^3 \right] = 253 \text{ lbs/in}^2$$

(g) Rearrange the interaction equation and determine the required resisting moment, Required M_r.

$$\text{Required } M_r = \frac{M_{max}}{1.33 - (f_a/F_a)}$$

$$= \frac{8872 \text{ ft-lbs}}{1.33 - [(26.3 \text{ lbs/in}^2)/(253 \text{ lbs/in}^2)]}$$

= 7236 ft-lbs

From table B-7 two 8 inch wide 12 inch deep stiffeners with 2-#6 bars furnish a resisting moment, Furnished M_r, as follows:

Furnished M_r = 2(3997 ft-lbs) = 7994 ft-lbs
Furnished M_r = 7994 ft-lbs > Required M_r = 7236 ft-lbs

...O.K.

(h) Check the combined axial and bending stresses as follows:

$$\frac{f_a}{F_a} + \frac{M_x}{M_{rm}}$$

$$\frac{26.3 \text{ psi}}{253 \text{ psi}} + \frac{8872 \text{ ft-lbs}}{7994 \text{ ft-lbs}} = 0.10 + 1.11 = 1.21 < 1.33$$

...O.K.

(4) *Summary.* A 12 inch by 16 inch wall stiffener with 4-#6 bars, as shown in figure 6-4, will resist the given loads.

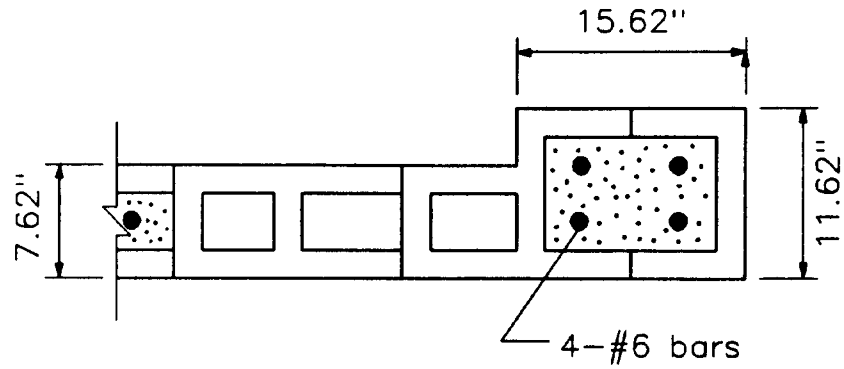

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Figure 6-4. Section A through wall stiffener.